Zipper Zeman Associates, Inc.
Geotechnical and Environmental Consulting
A Terracon Company

J-2367
May 30, 2006

MacLeod Reckord
231 Summit Avenue East
Seattle, Washington 98102

Attention: Mr. Terry Reckord

Subject: Report of Predesign Geotechnical Services
Burke-Gilman Trail Redevelopment
King County, Washington
King County Contract No. E53012E

Dear Mr. Reckord:

We are pleased to submit 3 copies of our report of geotechnical services for the Burke-Gilman Trail Redevelopment in King County, Washington.

Our services were completed in accordance with the work plan outlined in Attachment A of the Professional Services Agreement Contract between Macleod Record and ZZA (as amended by our Recommended Modifications to Work Plan letter dated May 9, 2006), and King County Contract No. E503012E. Preliminary results of this investigation were provided to you as information became available.

We appreciate the opportunity to provide geotechnical services on this project. Please contact us if you have any questions regarding this report or if we can provide assistance with other aspects of the project.

Sincerely,
Zipper Zeman Associates, Inc.

James P. Georgis, L.E.G. Project Geologist	James B. Thompson, Ph.D., P.E. Principal

18905 33rd Avenue West #117, Lynnwood, WA 98036 (425) 771-3304 Fax: (425) 771-3549

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Report of Predesign Geotechnical Services

Burke-Gilman Trail Redevelopment

King County, Washington

May 30, 2006

MacLeod Reckord

Landscape Architects

KING COUNTY

DEPARTMENT OF PARKS, CULTURAL AND NATURAL RESOURCES

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KING COUNTY, WASHINGTON

INTRODUCTION

This report presents the results of our predesign geotechnical services for the proposed Burke-Gilman Trail Redevelopment in King County, Washington. The planned trail redevelopment includes widening the asphalt surfaced portion of the trail to approximately 12 feet and providing 1-foot and 3-foot wide gravel shoulders on the left and right sides (looking upstation) of the trail, respectively.

The purpose of our services has been to observe surface conditions and review existing geologic and geotechnical literature relative to subsurface soil and groundwater conditions in the vicinity of the trail redevelopment area in order to formulate predesign geotechnical recommendations and criteria for use by others in schematic trail design and cost estimating. Our services included a literature review, site reconnaissance, geotechnical engineering analysis, and preparation of this report. These services were completed in accordance with the work plan outlined in Attachment A of the Professional Services Agreement Contract between Macleod Record and ZZA (as amended by our Recommended Modifications to Work Plan letter dated May 9, 2006), and King County Contract No. E503012E.

PROJECT DESCRIPTION

The trail corridor planned for redevelopment is about 2 miles long and located in portions of Sections 10, 11, and 15 of Township 26 North, Range 4 East in the City of Lake Forest Park. The southern end of the trail redevelopment (Station 0+00) is located at the boundary between the City of Seattle and the City of Lake Forest Park, near the east-west alignment of NE 145th Street. The northern end of the trail redevelopment (Station 104+40) is located near the west side of Log Boom Park. The approximate location of the trail redevelopment area is shown on the Site Vicinity Map, Figure 1.

The orientation of the Burke-Gilman trail within the redevelopment area varies from roughly north-south to approximately east-west making the description of site features relative to cardinal directions confusing. Therefore, we have described site features in terms of trail Station number and the feature's location right, center, or left of the trail alignment looking upstation. The location of the existing trail, surrounding site features and topography, and the trail redevelopment Station alignment are shown on Figure 2 (sheets L1.0 through L20.0).

In general, the existing asphalt trail within the redevelopment area is about 10 feet wide. The trail generally includes dirt shoulders and has discontinuous gravel shoulders up to about 2 feet wide. We understand that the trail redevelopment includes widening the asphalt surfaced portion of the trail to approximately 12 feet and providing 1-foot and 3-foot wide gravel shoulders on the left and right sides of the trail, respectively. We understand that the trail redevelopment may also include the following items in support of trail widening.

- Repaving or overlaying the existing 10 foot wide asphalt trail.
- Constructing new bridges to replace the existing McAleer and Lyon Creek pedestrian bridges.
- Replace existing retaining walls that are considered to be in poor condition and/or inadequate to support surcharge loads imposed by the new trail configuration.
- Construct additional retaining walls in new locations, as needed, to support cuts and fills associated with the new trail configuration.
- Install new culverts and/or modify existing culverts that cross the trail alignment.
- Implement trial protection and/or stabilization measures in areas of recent slope instability.

SITE CONDITIONS

GENERAL

ZZA completed a reconnaissance of the trail redevelopment area and immediate vicinity in April and May of 2006. Our reconnaissance included observations of surficial geologic conditions as well as existing trail, bridge, and trail-side retaining wall conditions. A summary of our observations is presented below.

SURFACE CONDITIONS

General

The Burke-Gilman trail is constructed on a former railroad embankment and is located a short distance from the northwestern shore of Lake Washington within the redevelopment area. The trail gradient is relatively flat and ranges from about elevation 30 to 36 feet. In the southern and northern portions of the alignment (Station 0+00 to 51+00 and Station 83+50 to 104+40), the embankment is located near the toe of steep to moderately steep slopes. In these areas, the embankment appears to be of side-cast construction, where the left portion of the alignment is cut into the slope and the right potion of the alignment consists of fill derived from the cut. The central portion of the alignment (Station 51+00 to 83+50) is located within a relatively flat alluvial valley and crosses McAleer Creek and Lyon Creek by means of pedestrian bridges.

The existing asphalt surface trail is about 10 feet wide and has discontinuous grass and gravel shoulders on one or both sides of the trail. In general, the width of the old railroad bed appears to range from about 11 to 18 feet. A system of drainage ditches is located along the left side of the trail and existing retaining walls are located on both sides of the trail, although the majority of the walls in close proximity to the trail are located on the right side.

Single-family residences are located on both sides of the trail along most of the alignment. In general, the houses on the right side of the trail are closer to the trail. The houses on the left side of the trail are typically constructed near the top of the moderately steep to steep slopes located along the left side of the trail in the southern portion of the trail alignment. Residential streets and driveways intersect and parallel portions of the trail alignment.

Existing site features including roads, residential structures, bridges, and retaining walls are shown on Figure 2. More detailed descriptions of existing retaining walls, bridges, trial conditions, areas of obvious slope instability, and wet soil conditions and surface water are presented below and in Tables 2 through 6.

Retaining Walls

There are approximately 13 and 42 existing retaining walls located on the left and right sides of the trail alignment, respectively. Existing wall types include rockery walls, cast-in-place concrete walls, timber pile walls, soldier pile walls, mechanically stabilized earth walls, modular block walls, timber crib walls, and railroad tie walls. Our reconnaissance included observations of the existing walls and an evaluation of wall conditions. A summary of our observations is presented in Tables 2 and 3.

McAleer Creek Bridge

The trail crosses McAleer Creek by means of a pedestrian bridge at about Station 67+50. The existing bridge is of steel construction with a concrete deck and is supported on concrete abutments. The bridge is about 12 feet wide and has a clear span of about 40 feet. The concrete abutments and bridge span appeared to be in serviceable condition with no obvious indications of distress. A high-flow bypass structure and rockery wall are located near the north side of the east bridge abutment.

Lyon Creek Bridge

The trail crosses Lyon Creek by means of a pedestrian bridge at about Station 78+15. The existing bridge span is constructed of timber with an asphalt surface. The bridge has a clear span of about 20 feet and is supported on driven timber piles. The timber piles appeared to be treated with creosote. In general, the visually observable portions of the bridge span and timber piles appeared to be in serviceable condition. The bridge approach fills are retained by timber lagging placed behind the driven timber abutment piles. The timber lagging appeared to be in serviceable condition at both abutments. However, the east abutment lagging has been undermined by creek scour and voids have developed within the approach fill.

Wet Soil Conditions and Surface Water

A system of drainage ditches is located along the left side of the trail over the majority of the alignment. These ditches appear to primarily receive water from overland flow, groundwater seepage, and from pipes and culverts servicing upgradient developed areas. Our reconnaissance included observations of wet soil conditions and surface water in the vicinity of the trail alignment. The reconnaissance disclosed numerous areas of wet soil and standing water along the alignment. The most extensive wet areas were located between Station 0+00 and 6+50 and Station 31+10 and 48+30. The southern area exhibited wet soil conditions, surface water within the trail side ditch, and obvious indications of groundwater seepage. The northern area also exhibited wet soil conditions and surface water in the trail side ditches. However, it appeared that a significant portion of the surface water in this area was conveyed to the ditches by existing pipes and culverts. A summary of our observations is presented on Tables 4 and 5. These tables include smaller areas of wet soil and surface water not discussed above.

Areas of Obvious Slope Instability

The existing trail was constructed on a former railroad embankment and the surrounding areas have undergone extensive development. The construction of the railroad embankment and development of the surrounding area have altered the original ground topography and vegetation. This alteration has eliminated many of the geomorphic features that could be used to assess past slope instability. However, our reconnaissance did identify two areas that exhibit obvious indications of past slope instability and one area that exhibits indications of past and recent instability.

Areas of past slope instability were observed on the left side of the trail between Station 6+80 and 16+60 and Station 91+00 and 92+80. Indications of past and recent slope instability were observed on the left side of the trail from Station 0+00 to about Station 6+80. Detailed descriptions of these areas are presented on Table 6 of this report.

Trail Condition

In general, the existing asphalt trail appears to be in serviceable condition. We did not observe any obvious or extensive areas of distress such as “alligator” cracking or en-echelon cracks that are sometimes indicative of yielding subgrade conditions or embankment instability, respectively. However, localized areas of distress interpreted as tree root damage were observed along much of the alignment. The observed root damage typically consisted of cracks and ridges in the asphalt surface.

GEOLOGIC CONDITIONS

General

The project site is located within the Puget Lowland near the northwestern shoreline of Lake Washington. The Puget Lowland is a north-south trending depression bounded on the east and west by the Cascade and Olympic mountain ranges, respectively. The topography and geology of the Puget Lowland are a direct result of several cycles of regional glaciation during the Pleistocene epoch. The most recent cycle of glaciation, known as the Vashon Stade of the Fraser Glaciation, ended approximately 13,500 years ago. The Vashon Stade is believed to have covered the Puget Lowland with up to 3,000 feet of glacial ice in the deeper portion of the Lowland.

Most of the Puget Lowland, including the project area, is underlain by a thick, complex sequence of Quaternary age sediments deposited by continental glacial advance and recession. These sediments overly Tertiary age bedrock of sedimentary and igneous origin. Sediments deposited during periods of glacial advance were densely compacted by the weight of the glacial ice. Looser, unconsolidated sediments were deposited during periods of glacial retreat.

Geologic conditions in the northern portion of the trail alignment were assessed by reviewing the *Composite Geologic Map of the Sno-King Area, Central Puget Lowland, Washington, 2004*. This map was produced by a joint effort between the Seattle-Area Geologic Mapping Project, the University of Washington, and the United States Geologic Survey. The *Geologic Map of the Edmonds East and part of the Edmonds West Quadrangles, Washington* (USGS Map MF 1541, 1983) was also reviewed relative to geologic conditions in the northern portion of the trail alignment. Geologic conditions in the southern portion of the trail alignment were primarily assessed by reviewing the *Geologic Map of Seattle, Washington* (USGS Open File Report 2005-1252). This report was prepared by the USGS in cooperation with the City of Seattle and the University of Washington.

Based on the mapped geologic conditions and our reconnaissance level site observations, we have divided the trail alignment into four geologic domains. A generalized description of the geologic conditions in each area is presented below.

Station 0+00 to 40+00

From Station 0+00 to about Station 40+00, the trail alignment is located near the toe of a moderately steep to steep east-facing slope. In general, the ground surface to the right of the trail embankment slopes gently towards Lake Washington. The steep slope to the left of the trail is primarily mapped as Quaternary age undifferentiated pre-Fraser sediments (Qpf). These deposits are described as interbedded sand, gravel, silt, and diamict (till). The deposits have been glacially overridden and are generally dense to hard in their undisturbed state. The gently sloping area to the right of the trial embankment is mapped as Quaternary lake deposits (Ql). These deposits are described as silt and clay with local sand layers, peat, and other organic sediments. The lake deposits were exposed by the lowering of Lake Washington around 1916.

Numerous indications of past slope instability and several areas of recent slope movement were observed on the steep slope on the left side of the trail in this area. Based on our observations, we anticipate that the mapped pre-Frasier deposits are overlain by colluvial soils of an indeterminate depth. The trail embankment (former railroad bed) appears to be of side-cast construction, where the left portion of the alignment is cut into the slope and the right potion of the alignment consists of fill derived from the cut. We anticipate that much of the trial embankment in this section is composed of fill and colluvium.

Station 40+00 to 51+00

From about Station 40+00 to about Station 51+00, the trail alignment is located near the toe of a moderately steep east-facing slope. In general, the ground surface to the right of the trail embankment slopes gently towards Lake Washington. The moderately steep slope to the left of the trail is primarily mapped as Quaternary recessional outwash (Qvr). The recessional outwash is described as moderately sorted to well sorted, stratified sand and gravel with some silty sand and silt. The outwash was deposited as the Vashon glacier retreated and has not been glacially overridden. The gently sloping area to the right of the trial embankment is mapped as Quaternary lake deposits (Ql). These deposits are described as silt and clay with local sand layers, peat, and other organic sediments.

The trail embankment (former railroad bed) appears to be of side-cast construction, where the left portion of the alignment is cut into the slope and the right potion of the alignment consists of fill derived from the cut. We anticipate that much of the trial embankment in this section is composed of recessional outwash and fill.

Station 51+00 to 83+50

From about Station 51+00 to about Station 83+50, the trail alignment is located within an area that slopes gently towards Lake Washington. Some moderately steep slopes are located to the left of the trail in the southern portion of this section. In general, this section of the alignment is mapped as Quaternary lake deposits (Ql) and Quaternary older alluvium (Qoal). The lake deposits are described as silt and clay with local sand layers, peat, and other organic sediments. The older alluvium is described as sand and gravel with some sandy, pebbly, organic rich silt. Portions of the trail embankment (former railroad bed) in this area are slightly higher then the surrounding area and probably includes some fill soils along with the mapped geologic units.

Station 83+50 to 104+40

From about Station 83+50 to about Station 104+40, the trail alignment is located near the toe of a moderately steep slope. In general, the ground surface to the right of the trail embankment slopes gently towards Lake Washington. The moderately steep slope to the left of the trail is primarily mapped as Quaternary older alluvium (Qoal). The older alluvium is described as sand and gravel with some sandy, pebbly, organic rich silt. The gently sloping area to the right of the trial embankment is mapped as Quaternary lake deposits (Ql). These deposits are described as silt and clay with local sand layers, peat, and other organic sediments.

The trail embankment (former railroad bed) appears to be of side-cast construction, where the left portion of the alignment is cut into the slope and the right potion of the alignment consists of fill derived from the cut. We anticipate that much of the trial embankment in this section is composed of fill and the mapped alluvial and lake deposits.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

It is our opinion that the proposed Burke-Gilman Trail Redevelopment is feasible from a geotechnical perspective. The following sections of this report present predesign geotechnical recommendations and criteria for use by others in schematic trail design and cost estimating. Our recommendations are divided into 7 primary categories which include; Geologic Hazard Areas; Preferred Trail Alignment; Trail Subgrade Considerations; Landslide Area Considerations; Retaining Wall Considerations; Bridges; and Culverts.

GEOLOGIC HAZARD AREAS

General

ZZA completed an evaluation of the surface conditions observed along the trail redevelopment area and published geologic maps and geotechnical reports for the project vicinity relative to Geologic Hazard Areas as defined in Chapter 16.16 *Environmentally Sensitive Areas* of the City of Lake Forest Park Municipal Code. Chapter 16.16 was adopted by the City on December 1, 2005 through Ordinance No. 930 and replaced Chapters 16.16 and 16.18 of the City Municipal Code. A summary of our findings and recommendations relative to Erosion Hazard Areas, Steep Slope Areas, Landslide Hazard Areas, and Seismic Hazard Areas are presented in the following sections.

Erosion Hazard Areas

Erosion Hazard Areas are defined by Chapter 16.16.040 of the City Municipal Code as those areas containing soils which, according to the USDA Soil Conservation Service, may experience severe to very severe erosion hazard, including slopes greater than 15 percent with exposed erodible soils. Soils which are considered by the City to be particularly susceptible to erosion include fill soils of virtually all soil types, loose sandy soils such as Vashon recessional outwash (Qvr), Esperance sand (Qe), weathered Vashon till (Qvt), and dense fine-grained clay (Qcl).

We reviewed the *Soil Survey of King County Area, Washington* (November 1973) and *Soil Survey of Snohomish County Area, Washington* (July 1993) prepared by the USDA Soil Conservation Service relative to mapped soil types along the trail alignment. The referenced publications do not include soil mapping or soil descriptions for the project area due to the developed nature of the area.

Our reconnaissance level observations and geologic map review suggest that the majority of the trail alignment and near surface soils on the adjacent slopes are primarily composed of fill, colluvium, recessional outwash, alluvium, and lake deposits. We therefore recommend that all areas disturbed by the trail redevelopment with slope inclinations greater than 15 percent be considered Erosion Hazard Areas. Development standards for Erosion Hazard Areas are presented in Chapter 16.16.280 of the City Municipal Code.

Based on our observations, it is our opinion that trail redevelopment within site Erosion Hazard Areas is geotechnically feasible provided that a temporary sediment and erosion control plan implementing Best Management Practices is developed to minimize erosion from disturbed areas with preventative measures. Preventative temporary erosion control measures may include, but are not limited to, silt fences, straw waddles, gravel check dams, sedimentation ponds, plastic sheeting on temporary construction slopes during wet weather, or other measures approved by the City planning department. It should be noted that development standards for Erosion Hazard Areas (Chapter 16.16.280 of the City Municipal Code) restrict site clearing between April 1 and September 30.

We recommend that permanent vegetation be established as soon as feasible and within one growing season. Permanent revegetation may include the use of hydroseed. Revegetation of permanent slopes steeper than 3H:1V may be enhanced through the use of rolled erosion control and root reinforcement materials such as Jute matting or Curlex II.

Steep Slope Hazard Areas

Steep Slope Hazard Areas are defined by Chapter 16.16.040 of the City Municipal Code as those areas not composed of consolidated rock with slope gradients of 40 percent or greater and with a vertical elevation change of at least 10 feet. Based on our observations and review of published geologic maps, it is our opinion that consolidated rock is not present near the ground surface along the trail alignment or on the adjacent slopes. We therefore recommend that slopes within and adjacent to the trail redevelopment area that exhibit inclinations of 40 percent or greater and at least 10 feet of relief be considered Steep Slope Hazard Areas. Development standards for Steep Slope Hazard Areas are presented in Chapter 16.16.310 of the City Municipal Code. Some of the development standards from Chapter 16.16.310 that appear most applicable to the trail redevelopment project are presented below. Please refer to Chapter 16.16.310 for a complete listing of development standards.

Section 16.16.310, Subsection A indicates that a minimum buffer shall be established at a horizontal distance of 50 feet from the top, toe, and along all sides of any slope 40 percent or greater. The buffer may be reduced to a minimum of 25 feet when a qualified professional demonstrates to the Planning Director's satisfaction that the reduction will adequately protect the proposed development, adjacent developments, and uses, and the Steep Slope Hazard Area.

Section 16.16.310, Subsection A, Number 2 indicates that removal of existing vegetation from a Steep Slope Hazard Area or buffer is prohibited unless otherwise provided for in an approved alteration plan.

Section 16.16.310, Subsection B, Number 2 indicates that approval of public and private trails may be allowed on steep slopes subject to compliance with recognized construction and maintenance standards. Construction of impervious surfaces, such as asphalt and concrete, that would contribute to surface water runoff, is prohibited unless the applicant demonstrates to the satisfaction of the Planning Director such action is necessary for soil stabilization or prevention of soil erosion.

Section 16.16.310, Subsection C, Number 1 indicates that alteration of slopes that are 40 percent or steeper with a vertical elevation change of up to 20 feet may be permitted provided that, a soils report prepared by a qualified professional satisfies the Planning Director that no adverse impact will result from the exception.

Section 16.16.310, Subsection C, Number 2 indicates that any slope that was created through legal grading activities may be regraded as part of an approved development plan; provided that, any slope that remains 40 percent or steeper following site development shall be subject to all requirements for steep slopes.

Section 16.16.310, Subsection D, indicates that when steep slope alterations are allowed under Section 16.16.310, subsections A through C, the proposal shall:

- Not decrease the slope stability on the site or on adjacent properties; and
- Be subject to certification by a qualified professional that the landslide hazard area can be modified safely or that the development proposal eliminates or mitigates the landslide hazard risk to the property or adjacent property; and
- Not adversely impact other sensitive areas, such as streams; and
- Not result in an increase in peak surface water flows or sedimentation to adjacent properties.

Given the conceptual nature of the project and the lack of a grading plan, it is not geotechnically feasible to fully address these Steep Slope Hazard concerns at this time. However, we anticipate that it will be feasible to meet the geotechnical requirements of Section 16.16.310, Subsection D during the design phase of the project.

In addition to the Steep Slope Hazard Area and buffer area exceptions outlined above, Chapter 16.16.220 of the City Municipal Code outlines projects and/or activities that are exempt from the regulations of Chapter 16.16. Chapters 16.16.230 through 16.16.260 outline additional exceptions to the development standards of Chapter 16.16 and allow work within sensitive areas and buffers. In particular, Chapter 16.16.230, Subsection 4 allows for activities within the improved right-of-way and Chapter 16.16.260 allows an exception for developments proposed by public agencies or public utilities.

We anticipate that some of the exemptions and/or exceptions presented in Chapters 16.16.220 through 16.16.260 may be applicable to the planned trail redevelopment. However, it appears that the Exemptions and Exceptions presented in the referenced sections of the Code are subject to interpretation and approval by the City Planning Director. We therefore recommend that the design team meet with the Planning Director early in the project to help define the standards to which the project will be subject.

Landslide Hazard Areas

Landslide Hazard Areas are defined by Chapter 16.16.040 of the City Municipal Code as a slope that is potentially subject to landslides. All Landslide Hazard Areas are classified as:

A. Class I: A slope that is less than 15 percent and is considered relatively stable;
B. Class II: A slope that is greater than 15 percent and is underlain by permeable soils that are relatively stable in their natural state but may become unstable if slope configurations or drainage conditions are modified;
C. Class III: A slope that is greater than 15 percent and is underlain by impermeable soils, and may be characterized by springs or seeping groundwater during the wet season.

Landslide Hazard Areas include Class II and Class III if any of the following are present:

1. Any area that has shown movement during the Holocene epoch (from 10,000 years ago to present) or which is underlain by significant waste debris of that epoch; or
2. An area potentially unstable on an alluvial fan or delta potentially subject to inundation by debris flows; or
3. An area with a slope of 40 percent or greater and with a vertical relief of 10 or more feet except an area composed of consolidated rock.

Our reconnaissance of the trail alignment included observations of groundwater seepage zones and site features suggesting past and recent slope instability. Based on our reconnaissance level observations and review of published geologic maps, we recommend that slopes within and adjacent to the trail redevelopment area from Station 0+00 to about 50+00 and from about Station 91+00 to about 92+80 with slope inclinations greater than 15 percent be considered Landslide Hazard Areas. This designation is primarily due to the interbedded nature of the mapped geologic unit, observed groundwater seepage, and our observations which suggest that the slopes may be mantled by significant deposits of colluvium. The extent of the recommended Landslide Hazard Area is in general accordance with the mapped Steep Slope and Landslide Areas presented on Map 4 of the City of Lake Forest Park Comprehensive Plan. Development standards for Landslide Hazard Areas are presented in Chapter 16.16.290 of the City Municipal Code. Some of the development standards from Chapter 16.16.290 that appear most applicable to the trail redevelopment project are presented below. Please refer to Chapter 16.16.290 for a complete listing of development standards.

Section 16.16.290, Subsection A indicates that a minimum 50 foot buffer shall be established from all sides of a Landslide Hazard Area. Buffer widths shall be extended or adjusted as needed to mitigate a steep slope or erosion hazard or to promote the health and safety of the public. The buffer may be reduced to a minimum of 25 feet when a qualified professional demonstrates to the Planning Director's satisfaction that the reduction will adequately protect the proposed development, adjacent developments, and uses, and the Landslide Hazard Area.

Section 16.16.290, Subsection B indicates that vegetation may not be removed from a Landslide Hazard Area or buffer unless permitted by a sensitive area permit.

Section 16.16.290, Subsection D indicates that permitted alterations to landslide hazards areas and buffers are only allowed as follows;

1. Landslide Hazard Areas located on slopes of 40 percent or steeper may only be altered if the alteration meets the standards and limitations established for Steep Slope Hazard Areas;
2. Alteration of Landslide Hazard Areas located on slopes less than 40 percent are permitted only under the following conditions and circumstances:
    A. The development proposal will not decrease slope stability on the site or on adjoining properties; and
    B. A licensed geologist or geotechnical engineer certifies that the Landslide Hazard Area can be safely modified or the development proposal designed so the landslide hazard risk to the property or adjacent properties is eliminated or mitigated;

C. The alteration will not adversely impact other sensitive areas, such as streams; and
D. The alteration will not result in an increase in peak surface water flow or sedimentation to adjacent properties.

3. Where such alterations are approved, buffers may not be required.

Given the conceptual nature of the project and the lack of a grading plan, it is not geotechnically feasible to fully address these Landslide Hazard concerns at this time. However, we anticipate that it will be feasible to meet the geotechnical requirements of Section 16.16.290, Subsection D during the design phase of the project.

In addition to the Landslide Hazard Areas and buffer area exceptions outlined above, Chapter 16.16.220 of the City Municipal Code outlines projects and/or activities that are exempt from the regulations of Chapter 16.16. Chapters 16.16.230 through 6.16.260 outline additional exceptions to the development standards of Chapter 16.16 and allow work within sensitive areas and buffers. In particular, Chapter 16.16.230, Subsection 4 allows for activities within the improved right-of-way and Chapter 16.16.260 allows an exception for developments proposed by public agencies or public utilities.

We anticipate that some of the exemptions and/or exceptions presented in Chapters 16.16.220 through 16.16.260 may be applicable to the planned trail redevelopment. However, it appears that the Exemptions and Exceptions presented in the referenced sections of the Code are subject to interpretation and approval by the City Planning Director. We therefore recommend that the design team meet with the Planning Director early in the project to help define the standards to which the project will be subject.

Seismic Hazard Areas

Seismic Hazard Areas are defined by Chapter 16.16.040 of the City Municipal Code as those areas underlain by low-strength fill and flood plain deposits with soil and groundwater conditions that are more susceptible to seismic hazard than other areas.

Our reconnaissance level observations and reviewed geologic maps of the area suggest that the majority of the trail alignment and near surface soils in the immediate vicinity from about Station 51+00 to 104+40 are primarily composed of fill, alluvium, and lake deposits. Given the low-lying nature of the trail in this portion of the alignment, we anticipate that near surface groundwater may be present and that these soils may be susceptible to liquefaction during a design seismic event. We therefore recommend that the alignment from about Station 51+00 to 104+40 be considered a Seismic Hazard Area. Development standards for Seismic hazard Areas are presented in Chapter 16.16.300 of the City Municipal Code.

PREFERRED TRAIL ALIGNMENT

Based on geotechnical site conditions observed during our reconnaissance of the trail redevelopment area and our review of readily available geotechnical reports and geologic maps, we have developed alignment recommendations for the proposed trail widening. Our recommendations are presented in Table 1 and are arranged into Station intervals. Please note that these recommendations are based on geotechnical considerations. We anticipate that the final trail location will be a compromise between geotechnical considerations and other factors including, but not limited to, the location of the trail easement, the location of adjacent private properties, the location of sensitive areas such as wetlands and streams, public input, and regulatory requirements.

TRAIL SUBGRADE CONSIDERATIONS

General

We understand that the trail redevelopment includes widening the existing 10 foot wide asphalt surfaced portion of the trail to approximately 12 feet and providing 1-foot and 3-foot wide gravel shoulders on the left and right sides of the trail, respectively. Predesign level recommendations for the trail redevelopment are presented below under the following headings: Earthwork Considerations; Drainage Considerations; and Construction Consideration.

Earthwork Considerations

General

Preliminary recommendations for use in the development of schematic trail layouts and cost estimates have been developed by ZZA based on our reconnaissance level site observations and limited literature review. Our recommendations are presented below under the following headings: Site Preparation; Structural Fill; and Permanent Cut and Fill Slopes.

Site Preparation

Site preparation should include the removal of all vegetation, root mass, organic soils, existing structures, and any deleterious debris from the planned trail alignment, or those locations where "structural fill" is to be placed. Preparation for site grading and construction should begin with procedures intended to drain ponded water and control surface water runoff. It will not be possible to successfully utilize on-site soils as "structural fill" if accumulated water is not drained prior to grading, or if drainage is not controlled during construction. Attempting to grade the site without adequate drainage control measures will reduce the amount of on-site soil effectively available for use, increase the amount of select import fill materials required, and ultimately increase the cost of the earthwork.

Following clearing and grubbing, organic-rich topsoil will need to be stripped along the planned trail alignment, as well as those areas to receive structural fill. The extent and thickness of topsoil within the trail redevelopment area is uncertain at this time and is expected to be variable. Any excavations that extend below finish grades should be backfilled with structural fill as outlined subsequently in this report. In our opinion, topsoil is not suitable for reuse as structural fill and should therefore be exported from the site or used for landscaping purposes.

After stripping of topsoil is completed, the exposed soils are anticipated to consist primarily of fill soils derived from variable parent geologic deposits. We anticipate that much of the fill will have moderate to high fines contents. After stripping, and prior to placement of structural fill, we recommend that pavement subgrade areas, and areas to receive structural fill be proofrolled and compacted to a firm and unyielding condition in order to achieve a minimum compaction level of 95 percent of the modified Proctor maximum dry density as determined by the ASTM:D-1557 test procedure. Due to the anticipated silty nature of the near-surface soils, proofrolling and adequate compaction can only be achieved when the soils are within approximately ± 2 percent of the optimum moisture content. Proofrolling should be accomplished with a heavy compactor, loaded double-axle dump truck, or other heavy equipment under the observation of a qualified geotechnical engineer. This observer will assess the subgrade conditions prior to filling. Areas where loose surface soils exist due to grubbing and stripping operations should be considered fill to the depth of the disturbance and treated as subsequently recommended for structural fill placement. The need for or advisability of proofrolling due to soil moisture conditions should determined at the time of construction. We recommend that a representative of our firm observe the soil conditions prior to and during proofrolling to evaluate the suitability of stripped subgrades.

Structural Fill

All fill material within the planned trail alignment should be placed in accordance with the recommendations herein for structural fill. Prior to placement, the surfaces to receive structural fill should be prepared as previously described. All structural fill should be free of organic material, debris, or other deleterious material. Individual particle size should be less than 6 inches in maximum dimension.

Structural fill should be placed in lifts no greater than 8 inches in loose thickness. The structural fill should be mechanically compacted to a firm and unyielding condition and to at least 95 percent of the modified Proctor maximum dry density as determined by the ASTM:D-1557 test procedure. We recommend that a qualified geotechnical engineer be present during grading so that an adequate number of density tests can be conducted as structural fill placement occurs. In this way, the adequacy of the earthwork may be evaluated as it proceeds. In the case of roadway and utility trench filling and wall backfilling in municipal rights-of-way, the backfill should be placed and compacted in accordance with current local codes and standards.

The suitability of soils for structural fill use depends primarily on the gradation and moisture content of the soil when it is placed. As the amount of fines (that soil fraction passing the U.S. No. 200 sieve) increases, soil becomes increasingly sensitive to small changes in moisture content and adequate compaction becomes more difficult, or impossible, to achieve. Generally, soils containing more than about 5 percent fines by weight (based on that soil fraction passing the U.S. No. 4 sieve) cannot be compacted to a firm, non-yielding condition when the moisture content is more than a few percent from optimum. The optimum moisture content is that which yields the greatest soil density under a given compactive effort.

At the time of reconnaissance, the near surface site soils appeared to have moisture contents above their optimum moisture content relative to their possible use as structural fill.

However, soil moisture conditions should be expected to change throughout the year. Consequently, use of the on-site soil as structural fill will require that strict control of the moisture content be maintained during the grading process. Selective drying of over-optimum moisture soils may be achieved by scarifying or windrowing surficial materials during extended periods of dry weather. Soils which are dry of optimum may be moistened through the application of water and thorough blending to facilitate a uniform moisture distribution in the soil prior to compaction.

In the event that inclement weather or wet site conditions prevent the use of on-site soil or non-select material as structural fill, we recommend that a "clean", free-draining pit-run sand and gravel be used. Such materials should generally contain less than 5 percent fines, based on that soil fraction passing the U.S. No. 4 sieve, and not contain discrete particles greater than 6 inches in maximum dimension. It should be noted that the placement of structural fill is, in many cases, weather-dependent. Delays due to inclement weather are common, even when using select granular fill. We recommend that site grading and earthwork be scheduled for the drier months.

Permanent Cut and Fill Slopes

In general, we recommend that permanent fill slopes be constructed no steeper than 2H:1V. However, fill slopes composed of fine grained soils should be no steeper than 3H:1V. If the slopes are exposed to prolonged rainfall before vegetation becomes established, the surficial soils will be prone to erosion and possible shallow sloughing. Surficial repairs, such as armoring affected areas with quarry spalls, may be necessary until vegetation is established.

When the ground surface slopes more than 5H:1V beneath proposed fills, the fill should be keyed and benched in suitable undisturbed native soils per the minimum requirements of the Uniform Building Code (UBC), Volume 1, Section 33.3.2, Preparation of Ground. We recommend that all benches be at least 8 feet wide and the key at the toe of the fill be at least 8 feet wide and 4 feet deep.

We generally recommend all permanent cut slopes be designed at a 2H:1V inclination or flatter. It has been our experience that permanent cut slopes steeper than 2H:1V will tend to ravel and slough to a flatter inclination over time. In addition, with the steeper slopes, topsoil erodes readily and it is more difficult and takes longer to establish vegetation for slope protection.

Permanent unsupported cuts into the toe of steep slopes located on the left side of the trail should be avoided, where feasible, particularly in the southern portion of the alignment. Cuts of this nature could result in destabilization of the slope.

Drainage Considerations

A system of existing drainage ditches is located along the left side of the trail over the majority of the alignment. These ditches appear to primarily receive water from overland flow, groundwater seepage, and from pipes and culverts servicing upgradient developed areas. Water collected in the ditches appears to cross the trail alignment within existing culverts to undisclosed discharge locations.

The trail redevelopment plan should include provisions to maintain positive drainage along the alignment to reduce the potential for overland flow across the trail and saturation of trail subgrade soils. Overland flow across the trail could result in the deposition of sediment on the trail and hazardous conditions for trail users. Saturated trail subgrade conditions could lead to premature pavement distress and migration of fines into trail base and gravel shoulder materials, which would decrease their support characteristics. Drainage measures could include:

- A drainage ditch system located along the left side of the trail similar to the existing system.
- Subsurface groundwater interceptor drains, and
- Below grade pipe culverts or surface ditches to direct collected surface water and hillside drainages across the trail.

We recommend that the design team consider the use of an open drainage ditch system along the left side of the trail similar to the existing drainage system. An open ditch drainage system is advantageous in that it effectively collects and conveys surface water and groundwater seepage, provides a temporary catchment area for small amounts of slide debris, provides some measure of water treatment and infiltration, and is easy to maintain using standard equipment and practices.

Subsurface groundwater interceptor drains effectively collect groundwater seepage and can collect surface water flow for some time if the drainage aggregate is extended to the ground surface. However, in our experience, drainage aggregate exposed at the surface tends to clog with sediment and organic debris and the ability of the system to collect surface water decreases over time. Maintenance of interceptor drains, particularly clogging of surficial aggregate, is very difficult to complete and typically requires partial or complete reconstruction of the system. We therefore recommend that subsurface interceptor drains be limited to the collection of groundwater seepage and not be used for the collection of surface water.

As mentioned above, trail subgrade strength and therefore the design life of the trail pavement, will depend on keeping the trail subgrade in a drained condition. We recommend that the bottom elevation of the drainage system be located at least 18 inches below the trail pavement, where feasible.

Collected water should be discharged at City approved locations. Water should not be discharged to Geologic Hazard Areas.

Construction Considerations

Earthwork may be difficult or impossible during periods of elevated soil moisture and wet weather due to the anticipated moisture sensitive nature of the site soils. Excavated site soils may not be reusable as structural fill depending on the moisture content and weather conditions at the time of construction. If soils are stockpiled for future reuse and wet weather is anticipated, the stockpile should be protected with plastic sheeting that is securely anchored. If on-site soils become unusable, it may become necessary to import clean, granular soils to complete wet weather site work.

Subgrade soils that become disturbed due to elevated moisture conditions should be overexcavated to expose firm, non-yielding, non-organic soils and backfilled with compacted structural fill. We recommend that the earthwork portion of this project be completed during extended periods of dry weather if possible. If earthwork is completed during the wet season (typically November through May) it may be necessary to take extra precautionary measures to protect subgrade soils. Wet season earthwork may require additional mitigative measures beyond that which would be expected during the drier summer and fall months. This could include diversion of surface runoff around exposed soils, draining of ponded water on the site, and collection and rerouting of groundwater seepage from upgradient on- and off-site sources. Once subgrades are established, it may be necessary to protect the exposed subgrade soils from construction traffic. Placing quarry spalls, crushed recycled concrete, or clean pit-run sand and gravel over these areas would further protect the soils from construction traffic.

Access along the existing asphalt surfaced trail with heavy construction equipment will need to be considered. Pavements along the existing trail may experience distress from construction traffic loads.

LANDSLIDE AREA CONSIDERATIONS

General

Our reconnaissance of the trail redevelopment area disclosed three areas exhibiting obvious surficial indications of past and/or recent landslide activity. A brief summary of our observations and recommendations is presented below.

Station 0+00 to 6+80

This portion of the alignment includes a near vertical upper scarp with about 5 to 15 feet of relief located above a colluvial slope ranging from about 15 to 50 degrees. The colluvial slope has about 15 to 25 feet of relief, pistol butted trees, and hummocky topography.

Recent earth movements were observed about 50 feet south of Stations 0+00 and near 4+60. The earth movement observed about 50 feet south of Station 0+00 appears to be located within the City of Seattle and south of the trail redevelopment area. The earth movement appears to consist of an earth slide in which soil from the upper scarp and from the colluvial slope moved down slope to the ditch located on the west side of the trail. It is unclear if the slide material extended onto the trail and was subsequently removed. The earth movement observed at Station 4+60 appears to consist of an earth topple in which clasts of hard silt with some fine sand and silty clay soils spalled from the upper scarp and toppled down the colluvial slope. Clasts of toppled soil were observed on the colluvial slope and in the trailside drainage ditch. Moderate groundwater seepage was observed near the base of the upper scarp and from the colluvial slope at numerous locations as outlined in Tables 4 and 5.

The steep slopes observed in this area appear to be more susceptible to slope instability than the remainder of the trail redevelopment area. This appears to be due to the interbedded nature of the soil deposits and widespread groundwater seepage. In general, this portion of the alignment is considered to be unstable and it is unclear where or when the next slope movement will occur.

We understand that the County would like to mitigate the potential for the recent landslide areas located within the redevelopment area to adversely impact the use of the trail, if feasible. Stabilization of the recent earth movement observed near Station 4+60 does not appear feasible because much of the unstable colluvial slope and the entire upper scarp appear to be located on private property and outside the trail right-of-way. A retaining structure completed within the right-of-way would not prevent the development of earth movements up slope of the structure. Furthermore, drainage improvements typically utilized to enhance slope stability, such as horizontal drains, due not appear well suited to this area due to the thinly interbedded nature of the soils and the tendency for horizontal drains to loose effectiveness over time.

The County may want to consider the construction of a catchment wall near the edge of the trailside ditch in areas of recent landslide activity. The wall would not stabilize the steep slope, but could be used to create a catchment area to collect and hold debris from small to moderate earth movements. This could reduce the need for emergency trail repairs and cleanup. Periodic cleaning of the catchment area behind the wall would be required to maintain the effectiveness of the system. It should be noted that large scale earth movement could damage the catchment wall.

We anticipate that catchment wall types could include gravity block walls, such as ecology or Ultra-Blocks, or soldier pile walls with timber lagging. A design phase geotechnical evaluation, including borings, would be required to develop design recommendations for a catchment wall system.

The recent landslide observed about 50 feet south of Station 0+00 appears to be located within the City of Seattle. It is our understanding that the trail redevelopment will not extend this far south.

Station 6+80 to 16+60

Moderately steep to steep slopes with somewhat hummocky topography and scattered pistol butted maple and alder trees were observed in this area. The slopes appear to be primarily mantled by colluvial soils. Indications of earth movement, interpreted as creep, were observed above the trail on the east side of NE 147th Street near Station 7+40. The observed indications of slope movement included leaning guardrails and tension cracks within the asphalt surface. It appears that the tension cracks have been sealed on several occasions. A timber crib wall interpreted to be in poor condition is located between the trail and the observed cracks in NE 147th Street. An area exhibiting slope morphology and vegetation maturity differing from the surrounding slope was observed near Station 12+00. This area is interpreted as an older landslide area.

In general, slope gradients in this area are flatter than between Stations 0+00 and 6+80 and there does not appear to be an extensive over steepened scarp near the top of the slope. Groundwater seepage was not observed in this area during our site reconnaissance. Based on these conditions, we anticipate that the potential for future landsliding in this area is lower that the previously described section from Station 0+00 to 6+80. As discussed in the previous section, a significant portion of the slopes located above the trail alignment appear to be located outside of the trail right-of-way and within private property. As such, stabilization of these slopes does not appear geotechnically feasible since a retaining structure completed within the right-of-way would not prevent the development of earth movements up slope of the structure. Furthermore, drainage improvements typically utilized to enhance slope stability, such as horizontal drains, due not appear well suited to this area due to the thinly interbedded nature of the soils and the tendency for horizontal drains to loose effectiveness over time.

Pavement distress was observed on the east side of NE 147th Street near Station 7+40 and a timber crib wall located between the trail and the distressed pavement area was observed to be in poor condition. It is unclear if this area is within the trail right-of way, the City street right-of-way, or both. The City and/or County may want to consider evaluating this area relative to stabilization considerations. We anticipate that this evaluation, if considered appropriate, could be completed as part of this project or separately.

Station 91+00 to 92+80

We observed severely leaning and overturned deciduous trees along the left side of the trail in this area. Groundwater seepage was also observed at the toe of the trail embankment along the northern side of Beach Drive Northeast. This area is interpreted as a possible landslide deposit. A relatively new soldier pile retaining wall is located along the southeastern side of Bothell Way NE in this area and may have increased the stability of the slope in this area by reducing driving forces.

We recommend that a design phase geotechnical evaluation of this portion of the alignment be completed, including geotechnical borings, to determine the stability of the trail embankment and provide recommendations for embankment stabilization if the calculated stability is considered unacceptable.

RETAINING WALL CONSIDERATIONS

General

Approximately 55 existing retaining walls are located along the trail alignment. Some of these walls could be affected by new loads associated with the trail redevelopment depending on the final trail layout. In addition to the existing retaining walls, we anticipate that new retaining walls will be needed on the right side of the trail to support fills placed to widen the roadbed and that new retaining walls may be needed on the left side of the trail to support cuts. Geotechnical considerations relative to schematic level project design and cost estimating for existing and new retaining walls are presented below.

Existing Retaining Walls

There are approximately 13 and 42 existing retaining walls located along the left and right sides of the trail alignment, respectively. The existing walls range from about 1 to 25 feet in exposed height and include rockery walls, cast-in-place concrete walls, timber pile walls, soldier pile walls, mechanically stabilized earth walls, modular block walls, timber crib walls, and railroad tie walls. These existing walls appear to be constructed within the trail right of way and on private property. Our reconnaissance included observations of the existing walls and an evaluation of wall conditions. A summary of our observations and our assessment of the condition of these walls is presented in Tables 2 and 3.

The majority of the existing retaining walls are of low to moderate height and are set back a reasonable distance from the current trail alignment. Most of these walls are not expected to be subject to significant load increases associated with trail redevelopment. However, our reconnaissance disclosed several rockery walls in the southern portion of the site that may be adversely affected by the trail redevelopment depending on the final trail location. Two walls that meet these criteria are located between Station 3+20 to 4+10 and Station 9+35 to 11+50.

We recommend that a design phase geotechnical evaluation of the existing retaining walls be completed once a trail layout and grading plan have been developed in order to identify walls of geotechnical concern. We anticipate that the design level effort will include several borings behind walls of concern to evaluate their stability relative to trail support. For cost estimating purposes, we recommend that a contingency be established for the replacement of existing retaining walls.

New Retaining Walls

We anticipate that new retaining walls will be needed to support fills on the right side of the trail and that new walls may be needed on the left side of the trail to support cuts. We understand that cast-in-place concrete walls are being used for schematic trail layout and cost estimating purposes. It is felt that this wall type can be constructed to the anticipated wall heights and will provide a reasonable representation of likely wall construction impacts and costs. It is likely that different types of retaining walls could be selected at specific locations during the design phase depending on an evaluation of site factors including the local soil and groundwater conditions, whether the wall supports a cut or fill, height of wall, backslope configuration, and foundation support considerations.

BRIDGES

General

Existing pedestrian bridges cross McAleer and Lyon Creeks at Stations 67+50 and 78+15, respectively. In general, the McAleer Creek Bridge is newer, wider, and in better condition than the Lyon Creek Bridge. We understand that the trail redevelopment will more likely than not include replacement of the Lyon Creek Bridge and may include the replacement of the McAleer Creek Bridge. A summary of pertinent site conditions in the vicinity of the existing bridges and geotechnical considerations relative to schematic level project design and cost estimating are presented below.

McAleer Creek Bridge

The McAleer Creek Bridge is constructed of steel with a concrete deck and is supported on concrete abutments. The bridge is about 12 feet wide and has a clear span of about 40 feet. The concrete abutments and bridge span appeared to be in serviceable condition with no obvious indications of distress.

As part of our literature review, we searched the GeoMap NW database for geotechnical reports completed in the vicinity of the bridge. Our search resulted in the following two geotechnical reports.

- *Geotechnical Evaluation, Burke-Gilman Trail Footbridge, Lake Forest Park, Washington, NCA File No.* 207497. This report was prepared by Nelson-Couvrette & Associates, Inc. and dated June 10, 1997. The report was completed for the McAleer Creek Crossing and includes four shallow hand excavated explorations in the vicinity of the existing bridge abutments. The explorations ranging from about 2.5 to 8 feet below the ground surface. In general, the exploration logs report loose to medium dense sand with variable silt and gravel content and soft silt with variable sand, gravel, and organic content. These deposits were interpreted as fill and alluvium. The report includes schematic drawings which appear to be consistent with the existing bridge configuration and recommendations for conventional shallow concrete abutment foundations.

- *Report*, *Geotechnical Engineering Services, McAleer Creek Bypass Pipeline, Lake Forest Park, Washington.* This report was prepared by GeoEngineers and dated April 23, 1993. The report includes the log of a geotechnical boring located about 20 feet east of the eastern bridge abutment. The boring extended to a depth of about 54 feet below the ground surface. The boring log reported about 2 feet of fill over alluvium consisting of interbedded loose to medium dense sand with variable silt content and soft to medium stiff silt to a depth of about 52 feet. Dense fine to medium sand was reported from 52 feet to the total depth explored at 54 feet below the ground surface. Groundwater was not explicitly reported on the log, but appeared to be about 8 feet below the ground surface based on soil moisture descriptions.

Based on conversations with MacLeod Record, we anticipate that the existing bridge might not be replaced. Based on our site observations, the existing bridge and concrete abutments appear to be in serviceable condition and functioning as intended under current and past loading conditions. However, if the bridge abutments are supported on shallow foundations as suggested by the Nelson-Couvrette report, the structure could be susceptible to unacceptable levels of seismic induced total and differential settlement.

If it is determined that the existing bridge will be retained, we recommend that a design phase effort be completed to determine if the existing abutments are supported on shallow or deep foundations. If it is determined that the structure is supported on shallow foundations, a design phase geotechnical study, including borings at the existing abutment locations, should be considered to determine the magnitude of potential seismic induced settlement. This information could be used to develop foundation underpinning recommendations if the calculated seismic settlements are considered unacceptable.

If it is determined that the existing bridge will be replaced, we recommend that a design phase geotechnical evaluation of the new bridge be completed once the abutment locations and loads have been identified. The design phase evaluation would include geotechnical borings, engineering analyses, and geotechnical foundation design recommendations.

For schematic planning and cost estimating purposes, we recommend that a contingency be established for foundation underpinning if the existing bridge is retained. We recommend that deep foundation support, such as auger-cast piles, be considered for a new bridge.

Lyon Creek Bridge

The Lyon Creek Bridge span is constructed of timber with an asphalt surface and is about 8.5 feet wide. The bridge has a clear span of about 20 feet and is supported on driven timber piles. The timber piles appeared to be treated with creosote. In general, the visually observable portions of the bridge span and timber piles appeared to be in serviceable condition. The bridge approach fills are retained by timber lagging placed behind the driven timber abutment piles. The timber lagging appeared to be in serviceable condition at both abutments. However, the east abutment lagging has been undermined by creek scour and voids have developed within the approach fill.

Our literature search did not yield any existing subsurface information in the vicinity of the Lyon Creek Bridge. However, the bridge is mapped within the same alluvial deposit as the McAleer Creek Bridge and our reconnaissance observations are in agreement with the mapped geologic unit. As such, we anticipate that the soils at the bridge location may consist of interbedded sand, silt, and gravel and may be susceptible to liquefaction and seismic induced settlement.

For schematic planning and cost estimating purposes, we recommend that deep foundation support, such as auger-cast piles, be considered for the new bridge. We recommend that a design phase geotechnical evaluation of the new bridge be completed once the abutment locations and loads have been identified. The design phase evaluation would include geotechnical borings, engineering analyses, and geotechnical foundation design recommendations.

CULVERTS

The existing trail is crossed by several culverts that appear to service the drainage ditch system located on the left side of the trail. Additional culverts are located parallel to the trail along the drainage ditch alignment where private driveways and public roads cross the trail. We anticipate that widening of the trail may require the installation of additional culverts at new locations and/or modification or replacing existing culverts.

At the time this report was prepared, the project was in the schematic phase and information regarding the location and depth of new culverts and which existing culverts may need modification was not available. We therefore recommend that a design phase geotechnical evaluation of new and modified culverts be completed once a final trail layout, grading plan, and drainage plan have been established. However, for schematic planning and cost estimating purposes, we anticipate that the installation of new culverts and the modification of existing culverts will be geotechnically feasible utilizing conventional construction practices, based on our reconnaissance level observations and literature review.

USE OF THIS REPORT

We have prepared this report for use by MacLeod Reckord, King County, and other members of the project team, for schematic design and cost estimating for the Burke-Gilman Trail Redevelopment. The data and report may be provided to prospective contractors for cost estimating purposes, but our report, conclusions and interpretations should not be construed as a warranty of surface or subsurface conditions.

Design phase geotechnical services, including subsurface explorations and analyses, should be anticipated to evaluate areas of geotechnical concern outlined in this report once a final trail layout has been established. Areas that may require subsurface explorations and additional geotechnical analysis include the McAleer and Allen Creek bridge abutment areas, areas of obvious past landslide activity, existing retaining walls that are in questionable condition and may affect the new trail, and new trail side retaining wall and catchment wall locations.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in this area at the time the report was prepared. No warranty or other conditions, express or implied, should be understood.

We trust that this report meets your present needs. Please call if you have any questions concerning the report.

PREFERRED TRAIL ALIGNMEI

Station	Preferred Direction of Widening *		
	Left	Center	Right
0+00 to 5+90		x	x
5+90 to 9+10		x	
9+10 to 15+80		x	x
15+80 to 16+60			x
16+60 to 27+10		x	x
27+10 to 31+60		x	x
31+60 to 33+10 **			
33+10 to 52+00		x	x
52+00 to 66+80 **			
66+80 to 68+10		x	
68+10 to 77+20 **			
77+20 to 78+20		x	x
78+20 to 80+90	x	x	
80+90 to 83+60 **			
83+60 to 102+80		x	x
102+80 to 104+40			x

* Preferred direction of trail widening within existing roadbe
** No Preference

TABLE 1
NT BASED ON OBSERVED GEOTECHNICAL SITE CONDITIONS

Comments
Steep unstable slope to left. Stay right of drainage ditch.
Moderately steep to steep slopes to left and right.
Stay right of drainage ditch.
Steep slope to left of trail. Slope does not appear to be accurately depicted on topographic plan. Significant retaining wall needed to shift trail left.
Stay right of drainage ditch.
Deep ditch to left on the order of 3 to 4 feet deep. Ditch appears to receive surface water from several culverts. Consider replacing ditch with buried pipe.
No obvious geologic constraints.
Moderately steep to steep slopes to left of trail with deep ditch and wet soil conditions/standing water in ditch.
No obvious geologic constraints.
McAleer Creek crossing area.
No obvious geologic constraints.
Lyon Creek to left.
Lyon Creek to right.
No obvious geologic constraints.
Stay right of drainage ditch.
Steep slope to left of trail approaching 70 degrees. Slope does not appear to be accurately depicted on topographic plan. Significant retaining wall needed to shift trail left.

ed looking upstation.

TABLE 2
EXISTING RETAINING WALL CONDITIONS
WALLS LOCATED TO LEFT* OF TRAIL

Station	Wall Type	Wall Height (ft)	Distance from Centerline of Existing Trail (ft)	Wall Condition	Comments
4+00 to 4+40	Rockery Wall	2	10 to 15	Poor	Overgrown by ivy and blackberry brush.
7+00 to 7+45	Timber Crib Wall	2 to 4	10	Poor	No obvious deflection. Moderate to severe wood rot. Guardrail posts on east side of NE 147th Street (upslope from wall) leaning east. Tension cracks in pavement
17+05 to 17+45	R/R Tie Wall	1.5	10 to 15	Good	
35+45 to 36+10	MSE Block Wall	20 to 25	30 to 35	Good	Relatively new wall with near vertical face.
38+45 to 40+85	CIP Concrete Wall	4 to 8	30 to 40	Good	
43+90 to 44+50	Timber Wall	3 to 4	25	Fair to Good	
48+65 to 48+90	Rockery Wall	5 to 6	20 to 30	Fair	Two tier rockery wall. Each tier is about 3 feet high.
67+25	CIP Concrete Wall	2 to 3	0	Good	Western McAleer Creek bridge abutment.
67+65	CIP Concrete Wall	2 to 3	0	Good	Eastern McAleer Creek bridge abutment.
67+60 to 68+00	Rockery Wall	3 to 5	3 to 15	Good	Wall extends below McAleer Creek bridge.
78+07	Timber Pile Wall	3 to 5	0	Fair	Western Lyon Creek bridge abutment. Creosote treated piles with timber lagging.
78+27	Timber Pile Wall	3 to 5	0	Poor	Eastern Lyon Creek bridge abutment. Creosote treated piles with timber lagging. Lagging undermined by creek scour.
85+00 to 96+00	Soldier Pile Wall	5 to 15	25 to 50	Good	Soldier pile wall along Bothell Wall NE with concrete panel lagging.

* Looking upstation

TABLE 3
EXISTING RETAINING WALL CONDITIONS
WALLS LOCATED TO RIGHT* OF TRAIL

Station	Wall Type	Wall Height (ft)	Distance from Centerline of Existing Trail (ft)	Wall Condition	Comments
0+00 to 0+35	Rockery Wall	2.5 to 4	30	Fair	
1+00 to 2+05	Rockery Wall	3 to 4	20 to 25	Fair	Rockery covered in ivy with poor exposure.
2+05 to 2+40	Rockery Wall	5 to 6	15	Fair	
2+40 to 2+80	Rockery Wall	3 to 4	15 to 20	Fair to Poor	Large gaps between facing stones.
2+80 to 3+20	Rockery Wall	5	20	Fair	Groundwater seepage from face of wall.
3+20 to 4+10	Rockery Wall	6 to 7	12 to 15	Fair to Good	Groundwater seepage from face of wall.
8+45 to 9+35	Rockery Wall	2 to 3.5	15 to 20	Fair to Good	
9+35 to 11+50	Rockery Wall	5 to 9	15 to 20	Fair to Good	Some large voids between facing stones. Relatively new wall.
19+60 to 22+65	Rockery Wall	1 to 3	10 to 15	Good	
24+05 to 24+70	Rockery Wall	2 to 3	15 to 20	Good	
24+70 to 25+40	Concrete Block Wall	3.5 to 4	12 to 15	Good	Wall backfill does not appear to be geogrid reinforced.
25+40 to 25+90	CIP Concrete Wall	3.5	12 to 15	Good	
27+60 to 27+95	Concrete Block Wall	3	15 to 20	Good	Wall backfill does not appear to be geogrid reinforced.
28+40 to 28+90	Rockery Wall	4 to 5	13 to 16	Fair	Some moderately weathered, fractured facing stones.
31+70 to 33+20	Brick & Stucco Wall	3 to 3.5	15 to 20	Good	
33+50 to 33+95	CIP Concrete Wall	2.5	20	Good	
33+95 to 34+90	Rockery Wall	2 to 3.5	10 to 20	Poor to Fair	Weathered rock with large voids between facing stones.
35+35 to 36+85	CIP Concrete Wall	4	10 to 17	Good	
36+85 to 37+25	Rockery Wall	2 to 3	20	Fair to Good	
37+25 to 38+15	Concrete Block Wall	2.5 to 5	20 to 25	Good	Unclear if wall backfill is geogrid reinforced.
38+25 to 38+65	Concrete Block Wall	2	25 to 30	Good	Wall backfill does not appear to be geogrid reinforced.
39+25 to 39+65	Rockery/Rubble Wall	2 to 3	30	Poor	Wall constructed of natural stone and concrete rubble.
40+25 to 40+90	Rockery Wall	1 to 2	22 to 25	Poor to Fair	
42+55 to 43+40	Rockery Wall	1 to 2	22 to 24	Fair to Good	
45+10 to 45+75	Concrete Block Wall	3 to 3.5	2 to 25	Good	Unclear if wall backfill is geogrid reinforced.

* Looking upstation

TABLE 3 (CONTINUED)
EXISTING RETAINING WALL CONDITIONS
WALLS LOCATED TO RIGHT* OF TRAIL

Station	Wall Type	Wall Height (ft)	Distance from Centerline of Existing Trail (ft)	Wall Condition	Comments
67+25	CIP Concrete Wall	2 to 3	0	Good	Western McAleer Creek bridge abutment.
67+65	CIP Concrete Wall	2 to 3	0	Good	Eastern McAleer Creek bridge abutment.
78+07	Timber Pile Wall	3 to 5	0	Fair	Western Lyon Creek bridge abutment. Creosote treated piles with timber lagging.
78+27	Timber Pile Wall	3 to 5	0	Poor	Eastern Lyon Creek bridge abutment. Creosote treated piles with timber lagging. Lagging undermined by creek scour.
84+65 to 85+65	Concrete Block Wall	2	20	Poor to Fair	Wall backfill does not appear to be geogrid reinforced. Western 40 feet of wall is destressed and overturning.
85+65 to 86+20	CIP Concrete Wall	2	20	Good	Exposed aggregate wall.
88+10 to 88+60	Concrete Block Wall/CIP Concrete	3.5	13	Good	1.5 foot high modular concrete block wall stacked on a 1.5 to 2 foot high CIP wall.
89+80 to 90+45	Concrete Block Wall	1.5 to 2	20	Good	Wall backfill does not appear to be geogrid reinforced.
90+45 to 91+80	Timber Wall	1	15 to 18	Poor	Railroad tie wall, distressed and moderately rotten.
95+60 to 96+00	Concrete Block Wall	2 to 2.5	15 to 20	Good	Unclear if wall backfill is geogrid reinforced.
96+00 to 96+85	CIP Concrete Wall	1 to 2	15 to 25	Good	Retaining wall for angled parking.
96+85 to 97+30	Concrete Block Wall	1.5 to 2	20	Good	Wall backfill does not appear to be geogrid reinforced.
97+30 to 97+85	Timber Wall	1.5	20	Good	No significant rot observed.
98+80 to 99+65	CIP Concrete Wall	2.5	18 to 20	Good	
99+65 to 100+20	Concrete Block Wall	2	20	Good	
100+20 to 101+35	Ecology Block Wall	2	18 to 20	Good	Modular concrete block planters on the order of 1 to 1.5 feet tall located above the ecology block wall.
101+35 to 101+90	CIP Concrete Wall	1.5	15	Good	
101+90 to 104+10	Ecology Block Wall	2	12 to 25	Good	Retaining wall for angled parking.

* Looking upstation

TABLE 4
WET SOIL CONDITIONS AND SURFACE WATER OBSERVED TO LEFT* OF TRAIL

Station	Observed Conditions	Distance from Centerline of Trail (ft)	Comments
0+00 to 6+50	Standing water in ditch & hydrophillic vegetation	7 to 40	Obvious groundwater seepage zones observed on steep slope to west of trail near stations 0+00, 1+00, 1+40, 6+00.
31+10 to 31+35	Flowing water in ditch	10	Water appears to be discharged from a drain pipe extended up-slope (west).
31+80 to 32+10	Wet soil and hydrophillic vegetation	20 to 30	
34+00 to 35+60	Standing water in ditch & hydrophillic vegetation	10	Discharge of about 1/2 gpm to ditch from up-slope pipe located at Station 39+25.
36+70 to 37+15	Standing water in ditch & hydrophillic vegetation	15 to 35	
37+40 to 37+85	Wet soil and hydrophillic vegetation	10 to 15	
38+25 to 41+40	Wet soil and hydrophillic vegetation	10 to 25	Discharge of about 1/2 gpm to ditch from up-slope pipe located at Station 44+80. Standing water in ditch near discharge pipe.
42+70 to 45+85	Standing water in ditch & hydrophillic vegetation	10 to 20	Ditch may receive surface water from pipe located at Station 46+60. No discharge observed at time of evaluation.
47+70 to 48+30	Flowing water in ditch & hydrophillic vegetation	10 to 15	Ditch receives surface water flow from up-slope property to west.
68+45 to 69+40	Standing water in ditch & hydrophillic vegetation	20 to 30	
72+35 to 73+00	Wet soil and hydrophillic vegetation	20 to 35	
85+80 to 88+05	Standing water in concrete lined ditch	11 to 13	
102+60 to 103+80	Wet soil	8 to 10	

* Looking upstation

TABLE 5
WET SOIL CONDITIONS AND SURFACE WATER OBSERVED TO RIGHT* OF TRAIL

Station	Observed Conditions	Distance from Centerline of Trail (ft)	Comments
0+40 to 0+65	Slight groundwater seepage and standing water	25 to 30	Seepage from toe of slope extending east of trail.
2+10 to 2+30	Wet soil at toe of rockery wall	20	
2+80 to 3+80	Flowing water at toe of rockery wall	18 to 25	Groundwater seepage observed from face of rockery wall between Station 3+15 and 3+60.
4+10 to 6+20	Standing water & hydrophillic vegetation	12 to 16	Standing water observed along west edge of Edge Water Lane and wet soils on slope between Edge Water Lane and trail.
91+10 to 91+80	Standing water with algae	17 to 26	Standing water in gravel parking area with well developed algae growth.

* Looking upstation

TABLE 6
SLOPE AREAS EXHIBITING OBVIOUS INDICATIONS OF PAST SLOPE INSTABILITY

Areas of past slope instability located to left* of trail

Station	Comments
0+00 to 6+80	This portion of the alignment includes a near vertical upper scarp with about 5 to 15 feet of relief located above a colluvial slope ranging from about 15 to 50 degrees. The colluvial slope has about 15 to 25 feet of relief, pistol butted trees, and hummocky topography. Recent earth movements were observed about 50 feet south of Stations 0+00 and near Station 4+60. The earth movement observed about 50 feet south of Station 0+00 appears to be located within the City of Seattle and south of the trail redevelopment area. The earth movement appears to consist of an earth slide in which soil from the upper scarp and from the colluvial slope moved down slope to the ditch located on the west side of the trail. It is unclear if the slide material extended onto the trail and was subsequently removed. The earth movement observed at Station 4+60 appears to consist of an earth topple in which clasts of hard silt with some fine sand and silty clay soils spalled from the upper scarp and toppled down the colluvial slope. Clasts of toppled soil were observed on the colluvial slope and in the drainage ditch. Moderate groundwater seepage was observed near the base of the upper scarp and from the colluvial slope.
6+80 to 16+60	Moderately steep to steep slopes with somewhat hummocky topography and scattered pistol butted maple and alder trees. The slope appears to be primarily mantled by colluvial soils. Indications of earth movement, interpreted as creep, were observed above the trail on the east side of NE 147th Street near Station 7+40. The observed indications of slope movement included leaning guardrails and tension cracks within the asphalt surface. It appears that the tension cracks have been sealed on several occasions. An area exhibiting slope morphology and vegetation maturity differing from the surrounding slope was observed near Station 12+00. This area is interpreted as on older landslide area.
91+00 to 92+80	We observed severely leaning and overturned deciduous trees. Possible colluvial mass. New soldier pile retaining wall located along southeastern side of Bothell Way NE may have increased stability of slope. Groundwater seepage observed on north side of Beach Drive NE in this area.

Areas of past slope instability located to right* of trail

Station	Comments
NA	No areas of obvious past slope instability were observed to the right of the trail.

* Looking upstation

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ZZA Zipper Zeman Associates, Inc.
Geotechnical and Environmental Consulting
A Terracon Company

J-2367
July 11, 2006

MacLeod Reckord
231 Summit Avenue East
Seattle, Washington 98102

Attention: Mr. Terry Reckord

Subject: Summary of Previous Geotechnical Reports
Burke-Gilman Trail Redevelopment
King County, Washington
King County Contract No. E53012E

Dear Mr. Reckord:

As requested, this letter provides a summary of the previous geotechnical studies which pertain to hazardous slopes along the segment of the Burke-Gilman Trail from the boundary between the City of Seattle and the City of Lake Forest Park on the south, and to the Logboom Park on the north. The letter also provides a summary of the conclusions and recommendations regarding hazardous slopes presented in the *Draft Report of Predesign Geotechnical Services* prepared by Zipper Zeman Associates dated May 30, 2006.

The following two previous geotechnical reports which deal with the hazardous slope issue have been provided to ZZA for review. These reports are summarized in this letter.

1. *Geotechnical Report, Burke-Gilman Trail Slides, Lake Forest Park, Washington, HWA Project No. 2000019-100, December 6, 2001, Revised February 18, 2002, prepared by HWA Geosciences, Inc.*
2. *Preliminary Geotechnical Investigation, Burke-Gilman Trail Redevelopment, NE 145th Street to Logboom Park, King County, Washington, HWA Project Nom 2005-027, April 15, 2005, prepared by HWA Geosciences, Inc*

The following two additional previous geotechnical reports pertain to the footbridge and bypass pipeline at McAleer Creek, not to the hazardous slope issue. These reports are **not** summarized in this letter.

1. *Geotechnical Evaluation, Burke-Gilman Trail Footbridge, Lake Forest Park, Washington, NCA File No. 207497*
2. *Report, Geotechnical Engineering Services, McAleer Creek bypass Pipeline, Lake Forest Park, Washington*

DECEMBER, 2001 HWA GEOSCIENCES REPORT

This previous study was completed to evaluate a landslide area located along the uphill side of the Burke-Gilman Trail in Lake Forest Park. The study was limited to the segment of the trail between NE 145th and NE 147th Streets.

The study focuses on eleven (11) individual areas which have slid in recent history and provides the following information for each slide area: station, plan dimensions and estimated depth, type of landslide, estimated recurrence interval, and preliminary cost estimates for mitigation. Anchored shotcrete was the only mitigation measure considered for the upper slopes. Drainage and regrade, excavate and replace, and wall and regrade were the three options considered for the lower slopes. The total estimated cost to mitigate the eleven (11) individual slides is dependent on the mitigation option selected , but generally appears to be on the order of several hundred thousand dollars for the lower slopes and on the order of one million dollars for the upper slopes.

The HWA study also commented on the condition of the slope segments in this area which have not slid in recent history. HWA states: " It is possible the areas that failed in the recent past are geologically different from those that have not. However, it is not obvious that this is true and it is also possible that the other areas just haven't slid in recent history. Therefore, it may be advantageous to do a similar treatment along the entire 500-foot reach rather than to remediate isolated slide areas". Although the HWA report does not provide a cost estimate to remediate the entire area, it is apparent that the cost would be significantly higher.

ZZA has the following additional specific comments regarding the December, 2001 HWA report.

- The anchored shotcrete option for the upper slope would consist of installing soil nails into the slope, placing reinforcing mesh, and shooting shotcrete (pneumatically placed concrete) on the slope. We have a concern regarding the feasibility of using this method of slope stabilization. This concern is related primarily to the interbedded nature of the soil deposits exposed on the upper slope face and the presence of groundwater seepage. Unless groundwater is adequately drained, water pressures tending to develop behind the anchored shotcrete could result in result in cracking or even collapse of the shotcrete facing.
- The estimated unit cost for the anchored shotcrete option seems to be low.
- Much of the upper slope area appears to be located on adjacent private property and not within the trail right-of-way. An easement would be required for work on adjacent private property.
- As a general rule, mitigation of the lower slope would have only a limited beneficial effect unless the upper slope is also stabilized . This is due to the fact that slide material from the upper slope would tend to collect on the lower slope and eventually move downward toward the trail.

- The drainage and regrade option for the lower slope would involve the installation of "finger drains" up the center of the slide area along with limited regarding of the lower slope. In our opinion, the drainage option would likely have only a limited effect on the stability of the lower slope due to the difficulty of intercepting ground water and adequately draining the slope. In addition, the "finger" drains would have to be relatively deep and would be difficult to construct considering the loose/weak nature of the colluvium and the presence of seepage in the trench excavation.
- The excavate and replace option would involve removal of the colluvium which mantles the lower portion of the slope, and replacement with select fill of higher strength. It is difficult to imagine how this work could be completed without potentially endangering uphill property. In addition, this option would represent a massive earthwork project.
- The wall and regrade option for the lower slope would involve the construction of a catchment wall along the uphill side of the trail along with limited regarding behind the wall. ZZA considers a catchment wall to be a viable option. The preliminary cost estimates prepared by HWA Geosciences, Inc. for the catchment wall are based on the use of a soldier pile wall scheme. In our opinion, other less costly wall types should also be considered.

ZZA also completed an evaluation of the slope area between NE 145th and NE 147th Streets. Numerous indications of past slope instability and several areas of recent slope movement were observed on the steep uphill slopes. We concluded that stabilization of this steep slope area would be difficult and expensive considering the soil and groundwater conditions, and the fact that critical upper slope areas are located outside of the trail right of way. As an alternative, the ZZA Draft report recommends that the County consider the construction of a catchment wall along the west aside of the tail to catch and hold debris from small to moderate earth movements.

ZZA noted that a timber crib wall interpreted to be in poor condition is located between the trail and NE 147th Street at about Station 7+40. Indications of earth movement, interpreted as creep, were observed at this location including leaning guard rails, and tensions cracks of the asphalt surface. . It is unclear if this area is within the trail right-of-way, the City street right of-way, or both. The City and/or County may want to consider evaluating this area relative to stabilization considerations

ZZA evaluated the steep slope are between NE 147th Street lope It is important to note the three additional steep slope areas have been evaluated by ZZA. The slopes in this area appear are flatter that between NE 145th and NE 147th Streets and there does not appear to be an extensive over steepened scarps near the top of the slope. Groundwater seepage was not observed in this area during our site reconnaissance. Based on these conditions, we anticipate that the potential for future landslides in this area is lower than for the segment between NE 145the and NE 147th Street.

ZZA observed severely leaning and overturned deciduous trees along the left side of the trail between Station 91+00 and 92+00. Groundwater seepage was also observed at the toe of the trail embankment along the north side of Beach Drive. This area is interpreted as a possible landslide deposit.. We recommend that the stability of this area be evaluated during the design phase geotechnical investigation.

APRIL 15, 2005 HWA PRELIMINARY INVESTIGATION

This previous study provides general comments regarding the proposed trail widening. The study notes that widening by cutting into the existing uphill slopes is complicated by two factors: 1) maintaining adequate drainage and 2) the potential for destabilizing the slopes downhill from existing homes or driveways. A number of different types of retaining walls are described in general terms. The use of soldier pile walls is suggested for taller slopes while gravity walls could be considered for shorter walls. The study suggests that soldier pile walls with possible tiebacks be considered for cuts in the existing slide area in the southern portion of this segment of the trail.

The information presented in the April 15, 2005 HWA preliminary investigation is generally consistent with the information presented in the ZZA *Draft Report of Predesign Geotechnical Services*. However, the ZZA report contains much more detailed information.

We trust that this letter provides the information needed at this time. Please contact us if you have any questions.

Sincerely,
Zipper Zeman Associates, Inc.

JAMES B. THOMPSON
STATE OF WASHINGTON
19409
REGISTERED
PROFESSIONAL ENGINEER
7/11/06

EXPIRES 04/0

James B. Thompson, Ph.D., P.E.
Principal